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Dynamically Coupled Percolation and Deformation Analysis of Earth Dams

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SYNOPSIS An effective stress method using two-dimentional finite elements is presented for predicting the generation and dissipation of pore water pressure coupled with deformation in saturated sands under earthquake shaking. The method has been used to analyze the post-earthquake slide of upstream slope of Shimenling earth dam in Liaoning province during Haicheng earthquake on Feb. 4, 1975.

INTRODUCTION

The concept of effective stress analysis for liquefaction of sands was first introduced by Finn et al. (1976, 1977), but the subject they treated is essentially an one-dimentional problem. The writer has worked in the same direction on two-dimentional problems using finite element method for undrained systems (Shen, 1978), and recently the method has been extended to including simultaneously the generation and dissipation of pore-water pressure coupled with permanent deformations by incorporating the Biot's consolidation theory into dynamic response analysis (Shen, 1980). It is believed that this method is especially suitable to studying the role of drainage in liquefaction of sands. An ill drainage condition may prevent the dissipation of excess pore pressure induced by earthquake motion and lead to a delay of slide of sand slop at some moment after shaking. An excellent example of this type of post-earthquake failure is the slide of upstream slope of Shimenling earth dam at about 80 minutes after the passage of main shock and it will be analyzed later using the proposed method.

ANALYTICAL PROCEDURE

The analytical procedure consists of earthquake stage and post-earthquake stage, and both stages again are subdivided into several time steps. It is necessary to carry out three different calculations in each time step at the first stage, i.e. dynamic response analysis based on equivalent viscoelastic model, calculation of the increments of residual volumetrical and deviatorical deformations and, finally, statically coupled percolation and deformation analysis based on Biot's consolidation theory for porous medium. The post-earthquake analysis, however, needs static calculation only.

The method of dynamic response analysis in time step Δt is similar to that presented by Idress et al. (1973) in their computer program QUAD-4, where the analysis is suppose to include the whole duration of seismic excitation. Since the shear modulus G and damping ratio D are assumed to be dependet upon the mean effective stress and shear strain amplitude, which always change from step to step, it is necessary to make the dynamic analysis iteratively.

The pre-earthquake static stresses are computed by means of non-linear secant modulus procedure, assuming the instantaneous application of overall gravity force and using hyperbolic stress-strain relationship for iteration. The static analysis is made in each time step when the increments of residual deformation are available from dynamic analysis. For this purpose, the increments of deformation are treated as initial strains, and a static consolidation problem of a porous system loaded by fictitious body force converted from these initial strains is solved. Eventually, we can obtain the development of pore water pressure and residual displacement of nodal points as well as the redistribution of the static stresses in soil elements. Finally, it is recommended to make a routine static slope stability analysis using slip circle method modified by A.W. Bishop in order to find out the critical moment and corresponding minimum factor of safty aginst stability failure.

CONSTITUTIVE RELATIONSHIP

The constitutive relationship for sands necessary for aforementioned analysis was presented elsewhere (Shen et al., 1980). A brief description is given below.

Basic assumption.- The deformation character of sands in any cycle of loading is determined mainly by the effective stress states in crest and trough of current loading cycle (i.e. the effects of all intermediate states of stress are neglected) as well as a stress history measure defined as accumulated shear straining.

If we denote stress in crest and trough of loading by superscript + and - respectively, then the corresponding mean normal and maximum shear stress in the plane strain condition may be expressed by

$$\sigma_m^+ = \frac{1}{2}(\sigma_1 + \sigma_3)^+, \quad \sigma_m^- = \frac{1}{2}(\sigma_1 + \sigma_3)^-$$
$$\tau_m^+ = \frac{1}{2}(\sigma_1 - \sigma_3)^+, \quad \tau_m^- = \frac{1}{2}(\sigma_1 - \sigma_3)^- \tag{1}$$

where σ_1 and σ_3 are the major and minor principle stress respectively (hereafter all stresses mean effective). Later, however, we will also use another set of parameters to describe a loading cycle

$$\sigma_s = \frac{1}{2}(\sigma_m^+ + \sigma_m^-), \quad \sigma_d = \frac{1}{2}(\sigma_m^+ - \sigma_m^-)$$
$$\tau_s = \frac{1}{2}(\tau_m^+ + \tau_m^-), \quad \tau_d = \frac{1}{2}(\tau_m^+ - \tau_m^-) \tag{2}$$

where σ_d and τ_d are known as amplitudes of cyclic normal stress and shear stress or, more precisely, amplitudes of mean normal stress and deviatorical stress. The measure of accumulated shear straining is defined as

$$\xi = \sum_{i=1}^{r} (\gamma_d)_i \tag{3}$$

where $(\gamma_d)_i$ is the cyclic shear strain amplitude of ith cycle of loading and r is the number of current cycle.

Coaction and asymmetry of cyclic stresses. - The number of parameters in eq. (2) may be reduced to three by means of normalization as

$$\bar{\sigma}_d = \frac{\sigma_d}{\sigma_s}, \quad \bar{\tau}_s = \frac{\tau_s}{\sigma_s}, \quad \bar{\tau}_d = \frac{\tau_d}{\sigma_s} \tag{4}$$

In order to better characterize a loading cycle, the shear strain amplitude γ_d will be used instead of τ_d, and two new measures named as coaction and asymmetry of cyclic stresses will be introduced.

The coaction measure of the cyclic normal stress with the cyclic shear stress is defined as

$$\omega = \bar{\sigma}_d (R^+ - R^-) \tag{5}$$

where $R^+ = \frac{\tau_m^+}{\sigma_m^+ \sin\phi}$ and $R^- = \frac{\tau_m^-}{\sigma_m^- \sin\phi}$ are the shear stress levels in crest and trough of loading respectively, ϕ is the friction angle of sand. ω is closely related to phase angle difference between cyclic normal and cyclic shear stress levels, and may change from maximum when angle difference β is zero to negative when $\beta > 90^\circ$. The asmymetry measure of cyclic stresses is defined as

$$\delta = \frac{R^+}{1-R^+} - \frac{R^-}{1-R^-} \tag{6}$$

δ is closely related to the normalized stress $\bar{\tau}_s$ in neutral state of loading and may reflect the effect of initial shear stress.

Furthermore, an equivelent number of uniform cycles for irregular loading is defined, based on the principle of equality of accumulated shear straining between irregular and uniform loadings

$$N_e = \frac{\xi}{(\gamma_d)_r} \tag{7}$$

where $(\gamma_d)_r$ is current shear strain amplitude.

Formulation of constitutive relations. - An experimental program of drained strain controled cyclic triaxial test on a medium sand was performed. The samples were consolidated under an axial to radial stress ratio $\left(\frac{\sigma_a}{\sigma_r}\right)_c$ from 0.5 to 3.0 and the cyclic axial stress $\pm\Delta\sigma_a$ was loaded maintaining the lateral stress σ_r =const. or mean stress $\frac{1}{2}(\sigma_a+\sigma_r)$=const. In addition, several tests of cyclic loading by pulsating all-round pressure only is also made. Based on the aforementioned assumption and experimental data, following sets of constitutive relations are proposed.

i). Dynamic shear modulus and damping ratio

$$G = \frac{1+k_3\omega}{1+k_1\gamma_d} k_2 \sqrt{\sigma_s} \tag{8}$$

$$D = D_{max} \frac{k_1\gamma_d}{1+k_1\gamma_d} \tag{9}$$

ii). Increments of volumetrical and deviatorical residual strains

$$\Delta\varepsilon_s = \left[c_1 \gamma_d \exp(-c_3\delta^2) + c_2 \frac{\sigma_d}{\sqrt{\sigma_s}}\right] \frac{\Delta N}{1+N_e} \tag{10}$$

$$\Delta\gamma_s = c_4 \delta (\gamma_d)^{c_5} \frac{\Delta N}{1+N_e} \tag{11}$$

iii). Static bulk and shear moduli

$$K = k_b \sqrt{\sigma_s} \tag{12}$$

$$G = k_d \left(1 - \frac{\tau_s}{\sigma_s \sin\phi}\right) \sqrt{\sigma_s} \tag{13}$$

where $k_1, k_2, k_3, c_1, c_2, c_3, c_4, c_5, k_b, k_d$ and D_{max} are material constants, ΔN is the number of cycles in time step Δt. It is natural to use the avarage values from these ΔN cycles for the dynamic variables γ_d, σ_d, ω and δ. The expressions for G and D originated from Hardin and Drnevich (1972) with modification by adding ω in eq. (8), which is introduced to take into account the coaction of normal with shear cyclic stresses, recalling that $k_1 = \frac{1}{\gamma_r}$ with γ_r as reference strain. The equations (12) and (13) are derived based on the assumption that the volumetrical compression and rebound curves of sands may be approximated by power function with power 0.5 and the shear curves under constant σ_s again by hypobolic function with initial tanget modulus $G_i = k_d\sqrt{\sigma_s}$ and ultimate shear stress $\tau_f = \sigma_s \sin\phi$. It is recommended to derive k_b from the reconsolidation test of samples, initially sheared by cyclic loading under undrained condition.

Based on the aforementioned constitutive relationships, a computer program-EFESD (Shen, 1980) has been developed with the dynamic response analysis section taken after the program QUAD-4.

ANALYSIS OF SHIMENLING DAM

On February 4, 1975, at 80 minutes after the passage of main shock of Haicheng earthquake of magnitude 7.3 in Rechter scale, suddenly took place a slide on the upstream

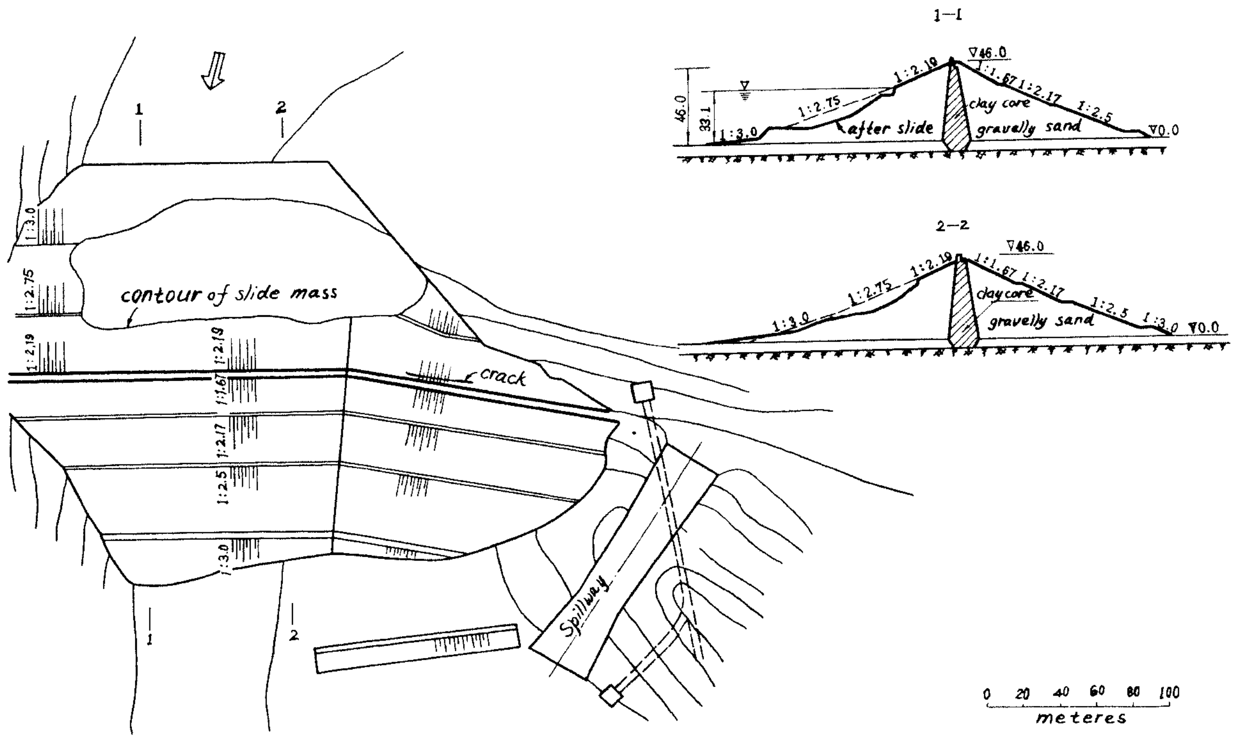

Fig.1. Slide of Shimenling Dam

slope of Shimenling earth dam, carried down 30000cub. meter of sandy material. Fortunately, the slip surface was rather shallow, and no catastrophe resulted (fig.1).

The dam has a maximum height of 46 m and a crest lenght of 338 m, and is located about 33 km south of the epicenter in the arear of intensity VII of modified Mercalli scale. The shell material consists of gravelly-sand with a large amount of clay fraction and had not been properly compacted during construction.

Since neither record of main shock at the dam site and nor experimental data of dam materials there are available, the following analysis takes qualitative character only in order to demonstrate the capacity of presented method for predicting the liquefaction potential of sands and evaluating the stability of earth structures both in earth quake and post-earthquake stages.

The dam profile is subdivided into 3 zone: zone 1 with the saturated sandy material in upstream shell, zone 2 with sandy material above the water level in upstream shell and in downstream shell and zone 3 with clay core. The finite element mesh contains 167 quadri-lateral elements and 188 nodes (fig.2). Each node has 3 degrees of freedom in zone 1 and 2 degrees in two orther zones.

The selected bed rock motion is one of after shock accelerograms in horizontal direction, recorded in HD station on Feb.15 (ML=5.4), which is scaled to a peak acceleration of 169 gal and predominant period of about 0.5 second (fig.3). The resonant period of this dam from preliminary elastic analysis is about 1.2 second. The whole duration of motion is subdivided into 8 steps with $\Delta t=2$ second in each step. The numerical integration of the equations of motion is worked out by using Wilson's θ-method with $\theta=1.4$ and a time increment of 0.025 second, and it

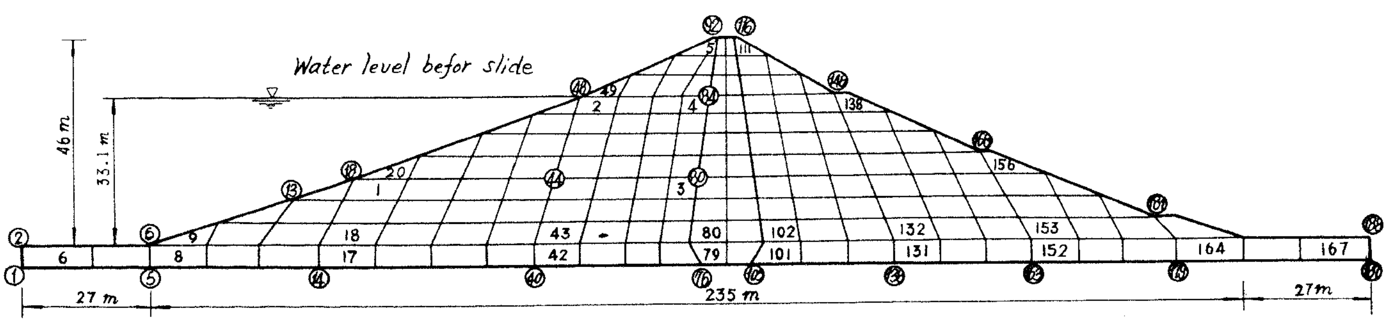

Fig.2. Finite Element Mesh

is repeated in 2 iterations for each step. The material constants used are selected referring to the experimental data of a sand from another site and have following values:

$k_1=3$, $k_2=0.6\times10^5$, $k_3=0.3$, $c_1=0.015$, $c_2=0.002$, $c_3=0.01$, $c_4=0.003$, $c_5=0.5$, $k_b=0.3\times10^5$ for loading, $k_b=1.5\times10^5$ for reloading, $k_d=0.1\times10^5$, $D_{max}=30\%$, $\sin\phi=0.75$, coefficient of permeability=0.01, saturated density=2300, moist density=2100, all in units SI, i.e. time in second, length in meter, mass in kirogramm and force in Newton. It is assumed that the core material has same constants, except $c_1=c_2=0$, $\sin\phi=0.3$ and cohesion=0.3×10^5. In this connection, σ_S in equations (8),(12) and (13) is calculated by

$$\sigma_S=\frac{1}{2}(\sigma_m^{+}+\sigma_m^{-}) + c\times\cot\phi$$

In the static analysis there may be two alternatives in establishing boundary condition for drainage in zone 1. In the first approach the upstream edge and water level in shell are described as equi-potential lines, i.e. there is full drainage allowing the excess pore water to escape through them. However, the lower part of slope surface is more like impervious because of sediment of finer particles on it in the period of three years operation of the reservoir. Furthermore, the water table in shell should rise when the pore water flows upward. No attempt has been made to take account of it in the computer program, but it is believed that an impervious boundary condition is more or less compatible with this circumstance in first approximation. Hence, in the second case of partial drainage the boundary parts between nodes 2 and 18 and that between nodes 48 and 84 are assumed to be impervious, whereas that between nodes 18 and 48 remains as equi-potential line. The remaining parts of the boundary are taken as impervious in both cases (fig.2).

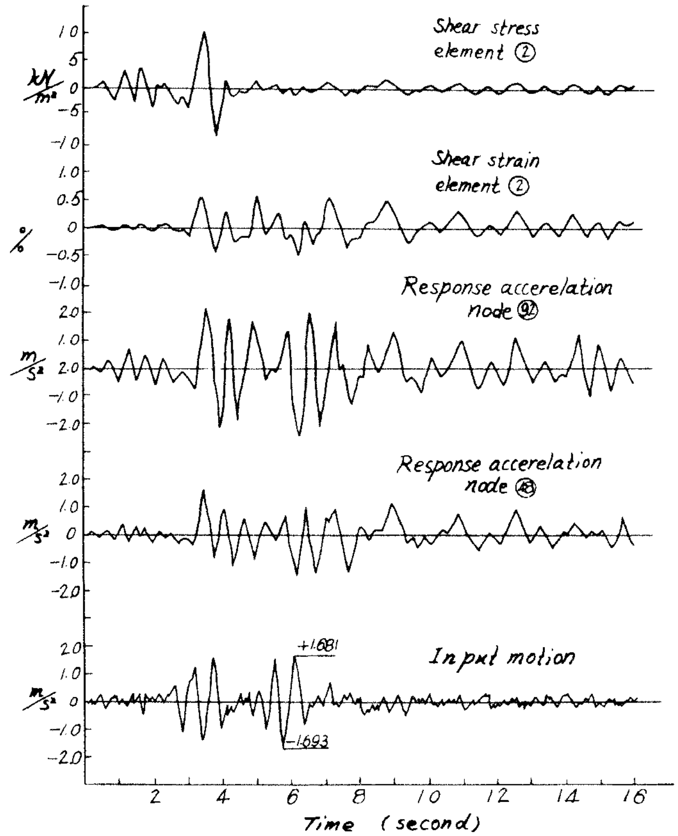

Fig.3. Input Motion and Computed Response

The results of dynamic response analysis are given in fig.3. Figure 4 illustrates the computed distribution of residual pore pressure and vectors of accumulated residual displacements at surface nodes at the end of earthquake motion for full drainage approach. Because of the absence of pore water in the downstream shell, vertical displacements in it are larger than that in the upstream shell, and on the contrary, horizontal

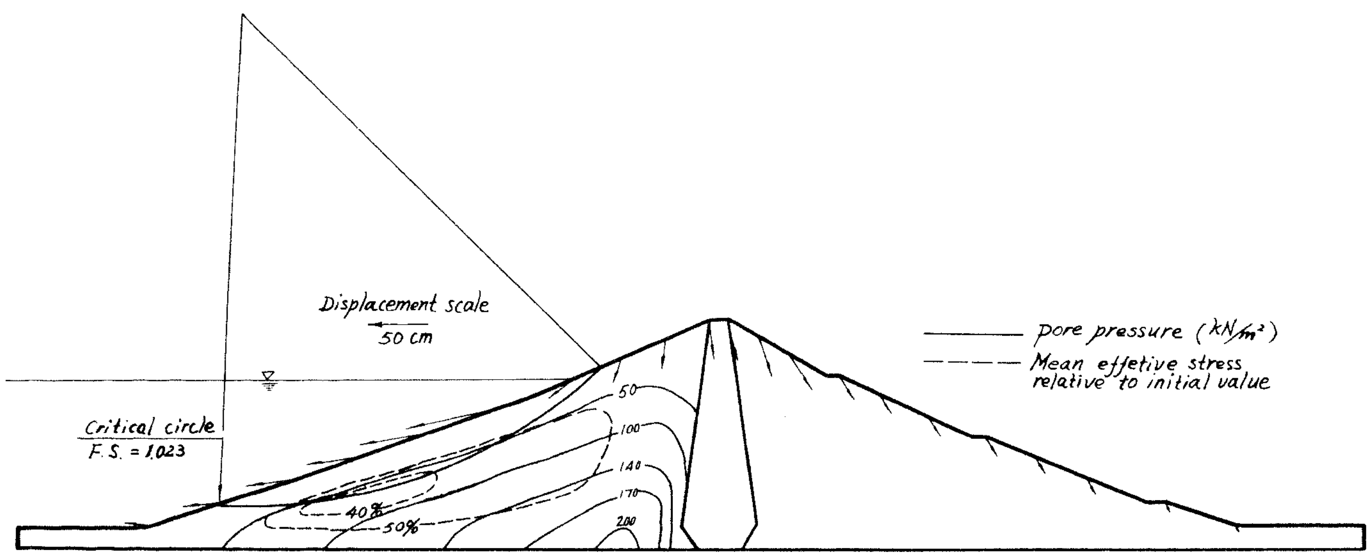

Fig.4. Residual Pore Pressure and Displacement Vectors (Full Drainage Approach)

displacements in the upstream shell are larger. There is not an element completely liquefied, but the mean effective stress is drastically dropped in some area and reaches the minimum 35% relative to its initial value (See dotted lines in this figure). When the drainage at the lower part of the slope is prevented as in case of the second approach , the residual pore pressure is accumulated near the boundary and causes some soil elements to liquefy (fig.5). Except this, the distribution of pore pressure is essentially identical with that in figure 4. It is evident from figure 5 that when pore pressure is dissipating in post-earthquake stage in the major part of upstream shell, it may increase in some time near the undrained boundary. This is evident also from fig.6.

The slope stability analysis has been carried out using Bishop's effective stress method with cohesion c=3 kN/m² and friction angle ϕ=38.6°. The introduction of a low value of cohesion for the sandy material is associated with consideration to avoid the critical slide surface being coincided with slope surface, and is compatible also with relative heavy content of finer particles in this sand. It also needs to point out that the static value sin38.6°=0.623 is different from sin ϕ=0.75 used in the dynamic analysis previously. The later is in 20% larger than the first.

The results of stability analysis are shown also in figures 4 and 5, from which it is clear that the critical slip circles always pass through weakest zones, where the pore pressures reach higher values relative to initial effective stresses. Figure 5 also illustrates, though qualitatively, that in case of the partial drainage the slide may delay to some moment after the passage of quake motion as happend actually. The extraordinarily low values of factor of safty are believed due to the assumption of perfect impervious boundary in one hand and low cohesion of soil in the other hand. The rapid dissipation of excess pore pressure suggests that the adopted coefficient of permeability may be too higher considering the existance of the large amount of finer particles in the actual soil.

Finally, it may be concluded from theoretical consideration, that if the sandy material of dam shell is ideally homogeneous and the slop surface is absolutely pervious, the most dengerious instant for stability must be at the end or in the stage of earthquake motion, when the overall excess pore pressure distribution reaches its maximum. An important factor contributed to the delay of slide as happend in Shimenling dam is believed to be associated with the deposition of less pervious material on the outer parts of the dam shell. The results of analysis emphasize the importance to guarantee the high quality of drainage as well as the high quality of compaction of sandy materials in design and construction of earth structures to bear earthquake shaking.

CONCLUSION REMARKS

There are two kinds of constitutive models commonly used in soil dynamics, i.e. elasto-plastic models and visco-elastic models. It is well known that deformation of sands is elasto-plastic in nature and hysteretic damping predicted by elasto-plastic models is more plausible than viscous damping by visco-

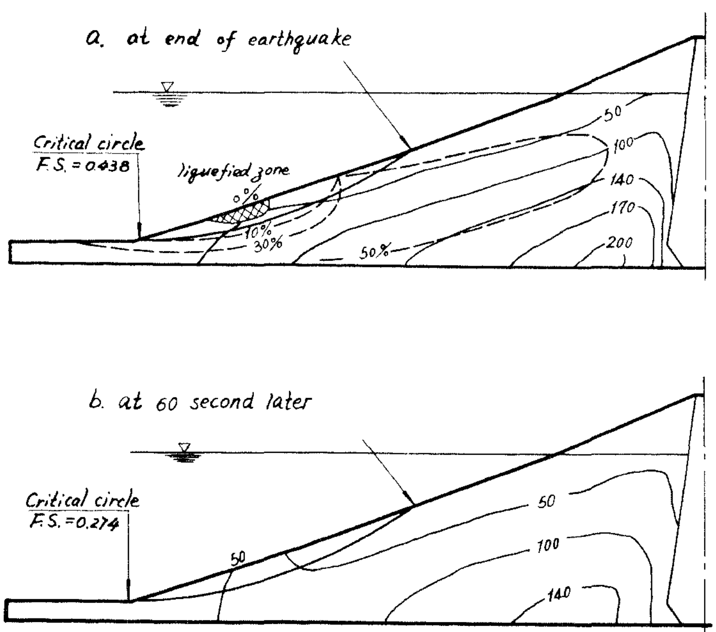

Fig.5. Pore Pressure Redistribution and Critical Slip Circles for Partial Drainage Approach

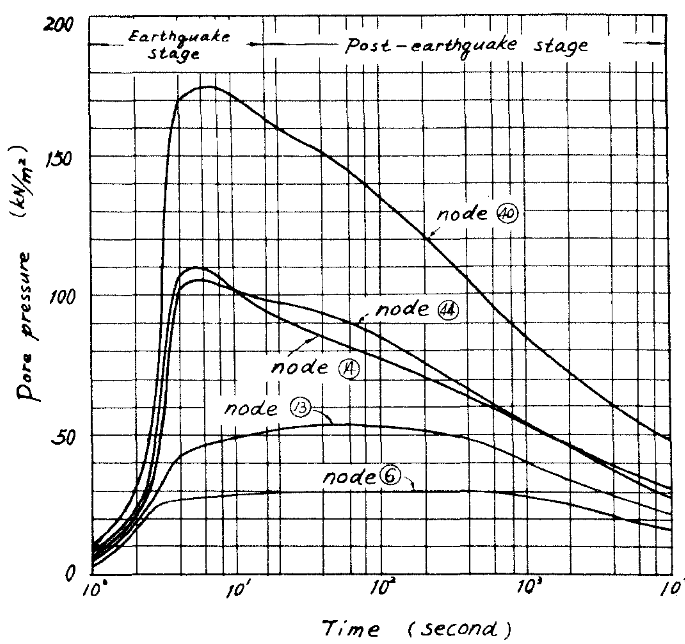

Fig.6. Time History of Pore Pressure at Selected Nodes

elastic models. But at the present time the generally accepted elasto-plastic model for dynamic analysis of soils has not been elaborated. Furthermore, it would be very costly and troublesome to use such a model in engineering practice at current stage of development of computer technique. Hence, It is the writer's opinion that the analytical procedure based on visco-elastic model originally developed by Berkeley school, University of California, is adequate for practical use, of cause one must keep in mind that the higher frequency components of response stress and strain may be damped out in calculation using this model.

On the other hand, the routine procedure for evaluating seismic stability of earth slopes by introducing an inertial force exerting upon the potential sliding mass seems too simple to characterize their actual failure under earthquake excitation. However, there is no evidence of any important role of inertial force in failure of earth slopes, and none of slide failures during earthquake has been reported if the slope is composed of unsaturated soils. It may be concluded with confidence that any severe failure of soil slopes under seismic excitation is associated with pore pressure build-up in them, Therefore, the effective stress approach is a reasonable step forward in seismic design of earth structures.

In this paper, the analytical procedure based on visco-elastic model is further improved by incorporating the effective stress concept and stepwise increment of mean value of residual strains. Preliminary application of this procedure to Shimenling dam and other earth structures already shows its general usefulness, although there still remains lot to be done in refining the concrete expressions of constitutive relationship as well as in establishing the corresponding experimental methods both in laboratory and field to obtain the necessary soil parameters.

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